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GUIDELINES FOR ASSESSMENT OF WAVE EFFECTS ON BRIDGES

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CONTENTS

S. No.	Description	Page No.
Personnel of the Bridges Specifications and Standards Committee		i
Symbols		iii
Definitions		iv
Introduction		v
1	Scope and Applicability	1
2	Hydrodynamic Effects on Bridges	1
3	Design Wave Parameters	1
4	Deck Level Identification	2
	4.1 Tidal Range	3
	4.2 Storm Surge	3
	4.3 Wave Crest Height	3
5	Wave Transformation	3
	5.1 Wave Refraction	3
	5.2 Wave Shoaling	4
	5.3 Wave Breaking	4
6	Wave Regime Classification	4
7	Wave Forces on Small Diameter Circular Members	5
	7.1 Non Breaking Waves	5
	7.1.1 Wave Kinematics	5
	7.1.2 Morison Theory	6
	7.1.3 Influence of Currents	7
	7.2 Breaking Waves	8
8	Wave Forces on Large Diameter Members	8
9	Wave Forces on Reflective Structures	8
	9.1 Goda's Method	9
10	Numerical Modelling for Wave Forces	10
11	Load Combinations	11
Example Problem 1	Wave Force on a Pile using Morison's Equation	12
Example Problem 2	Wave and Current Force on a Pile Using Morison's Equation	14
Example Problem 3	Breaking Wave Forces on a Pile	15
Example Problem 4	Minimum Deck Level Fixation for Bridges	16
Example Problem 5	Wave Force on a Reflective Stricture Using Goda's Method	17
Flow Chart: Estimation of Wave Forces		20
References		21
Appendix-I	Wave Parameters at Various Locations	22
Appendix-II	Tidal Range along India Coast	24
Appendix-III	Storm Surge	25

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SYMBOLS

H	Wave height (m)
L	Wave length (m)
T	Wave period (s)
d	Water depth (m)
D	Characteristic structure dimension (m)
g	acceleration due to gravity (m/s^2)
γ_w	Unit weight of water (N/m^3)
ρ	Density of water (kg/m^3)
k	wave number (m^{-1})
ω	angular frequency of wave (rad/s)
H_b	breaking wave height (m)
C	wave celerity (m/s)
C_g	group velocity (m/s)
u	water particle velocity in horizontal direction (m/s)
v	water particle velocity in vertical direction (m/s)
F_D	Drag force (Nm/s^2)
F_I	Inertia force (Nm/s^2)
C_D	Drag coefficient
C_M	Inertia coefficient

DEFINITIONS

Wave Height (H):

Vertical distance between the wave crest and adjacent trough

Wave Length (L):

Horizontal distance between two successive crests (or troughs)

Wave Period (T):

The time it takes for a wave to move a distance of one wavelength

Significant Wave Height (H_s):

Average height of the highest one-third of all waves measured

Deep Water Waves:

Waves moving through water with ratio of depth (d) to wavelength (L) are more than 1/2.

Shallow Water Waves:

Waves moving through water with ratio of depth (d) to wavelength (L) are less than 1/20.

Intermediate Water Waves:

Transition region between deep and shallow water where ratio of depth to wavelength is less than 1/2 and more than 1/20 (1/2 > d/L >1/20)

Highest Astronomical Tide:

It is the highest level which can be predicted to occur under average meteorological conditions and any combination of astronomical conditions. It is the highest high tide.

Lowest Astronomical Tide:

It is the lowest level which can be predicted to occur under average meteorological conditions and any combination of astronomical conditions. It is the Lowest Low tide.

Maximum Tidal Range:

Vertical difference in height between Highest Astronomical Tide and Lowest Astronomical Tide.

Still Water Level:

The level of the sea surface without considering waves but including tides and storm surges.

Charted Datum:

The datum from which water depths are measured in hydrographic (nautical) charts. Usually it is taken as Lowest Astronomical Tide.

Storm Surge:

Storm surge is the abnormal rise in seawater level during a storm, measured as the height of the water above the normal predicted astronomical tide. The surge is caused primarily by a storm's winds pushing water onshore.

INTRODUCTION

Coastal bridges and sea links are subjected to severe wave loads, especially during extreme conditions such as cyclones. The severity of these incidences ranges from small visible cracks to total collapse of the structure. Estimation of wave forces on bridge piers and determination of deck level are key aspects for safe and reliable design of coastal bridges.

Considering the expertise of National Institute of Ocean Technology (NIOT), Indian Road Congress had inducted Dr. M.V. Ramana Murthy, Director, NCCR and Group Head, Ocean Structures, NIOT, as a corresponding member of B-2 Committee on Loads and Load Combinations and requested NIOT to prepare Guidelines for the Estimation of Wave Pressure on Bridges. Accordingly, NIOT under the guidance of Dr. M.V. Ramana Murthy had initially prepared the detailed guidelines for estimation of various wave forces acting on the bridge piers and different scenarios to be considered by the engineers and designers for design of bridge piers and foundations in marine condition and presented this document to the B-2 Committee and the B-2 Committee felt that the information contained in the document is vast and recommended constitution of a sub-committee consisting of Shri A.K. Banerjee, Dr. M.V. Ramana Murthy, Dr. Mrs. Lakshmi Parameswaran, Dr. D.K. Kanhere, Shri P.G. Venkataraman, Dr. Harshavardhan Subbarao to review the document for finalization by B-2 Committee.

The Sub-committee conducted three meetings at NIOT, Chennai and critically reviewed the document and suggested modifications in the content and document flow for improvement of the guidelines to make it more convenient for designers to implement in their designs. Based on the recommendations suggested by the Sub-Committee, the document was revised and presented to the main B-2 Committee for review. Final document was prepared based on B-2 Committee recommendations and comments and presented by NIOT in the 14th meeting of B-2 Committee on 30th May, 2020. The B-2 Committee approved the guidelines in its 14th meeting and recommended for further processing.

The draft document was considered by the Bridges Specifications and Standards Committee (BSS) in its meeting held on 27.10.2020 and approved the document with certain suggestions. The draft was modified as per suggestions of BSS Committee, Subsequently, the draft was approved by the Executive Committee on 20-02-2021. The draft was discussed and approved by the Council of the IRC in its 221st meeting held on 20-02-2021 at India Internationals Centre New Delhi.

The Composition of Loads and Load Combination Committee (B-2) is as given below:-

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1 SCOPE AND APPLICABILITY

The objective of the document is to provide general guidelines for design engineers on hydrodynamic effects experienced by bridges in Open Ocean and commonly accepted methods for estimation of wave forces. Identification of deck level and estimation of forces on bridge pier are the key aspects for safe and reliable design of coastal bridges. In coastal regions increasing wave nonlinearity and wave-structure interaction play a fundamental role resulting in extreme complex pattern and local concentration of wave energy along the structure. This guideline provides a framework for estimation of wave and current forces on various bridge elements. However, forces due to rare events like tsunami are not covered in this guideline.

This publication is meant to serve as a guide to both the design engineer and the construction engineer but compliance with the rules therein does not relieve them in any way of their responsibility for the stability and soundness of the structure designed and erected by them. Estimation of hydrodynamic forces on bridges require an extensive and thorough knowledge of the science and technique involved and should be entrusted only to specially qualified engineers with adequate practical experience in Marine engineering and capable of ensuring careful execution of work.

2 HYDRODYNAMIC EFFECTS ON BRIDGES

Coastal bridges are subjected to extreme loads during cyclones. The variety of damage includes failure of shear key, displacement of span from their original position, spans being washed away from support piles, tilt and collapse of piles and excessive cracks in bulky decks and girders. The severity of these incidences ranges from small visible cracks to total collapse of structure.

3 DESIGN WAVE PARAMETERS

The design wave parameters required for the estimation of wave forces are the wave height and wave period **(Fig.1)** of the representative wave for a specified return period. If the dynamic response is significant then a possible range of wave heights and periods should be considered and the combination with the greatest dynamic amplification should govern the design and dynamic analysis may be carried out. If design wave parameters are produced by cyclones then the most severe probable cyclone path and the corresponding wave height and period at the site for this path should govern the design.

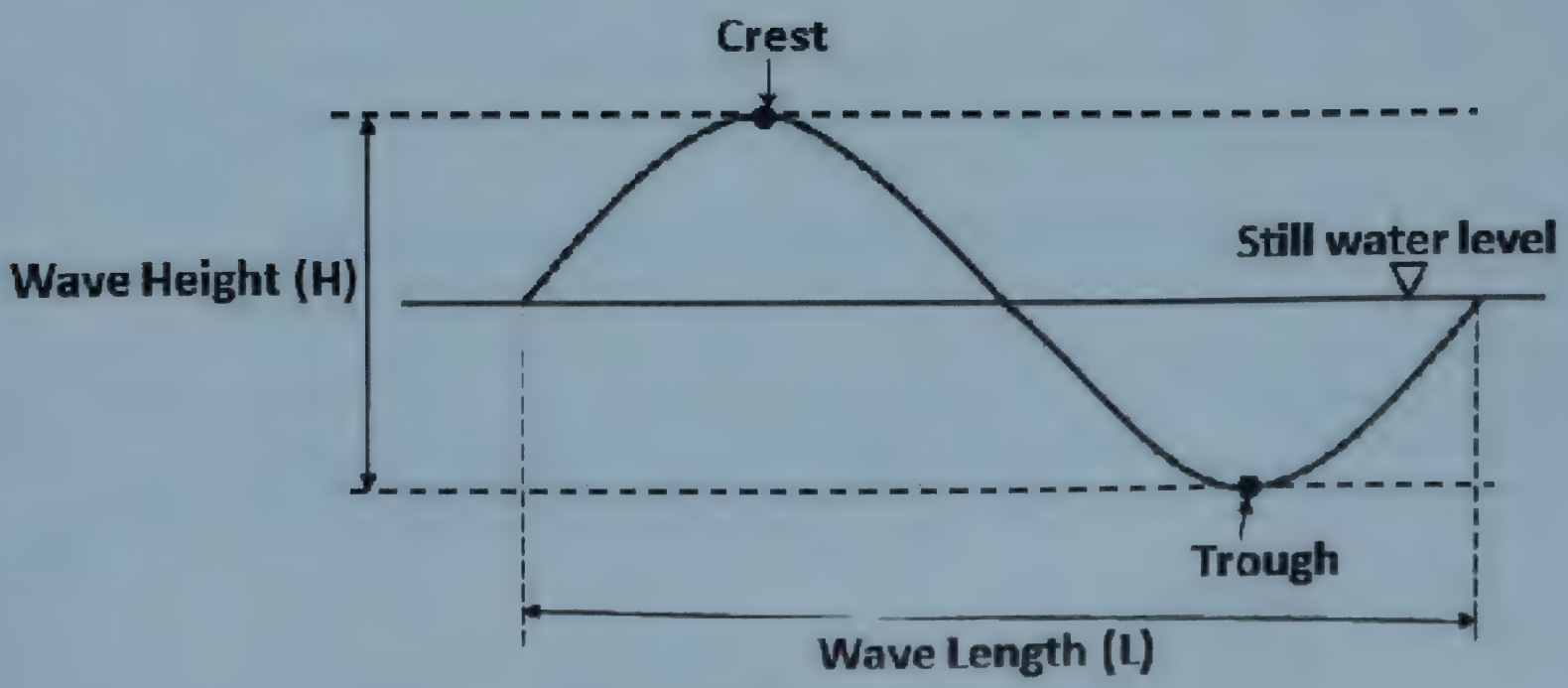


Fig.1 Design Wave Parameters

Wave Heights for various return periods at 25 m water depth and predominant wave angle at various locations along Indian coast are provided in the **Appendix-I & Appendix-IA**. These details can be used for preliminary design. However, for detailed design it is recommended to carry out detailed site specific measurements along with numerical modeling to produce hindcasts for determining the design parameters.

Some of the important additional wave parameters which may be required for estimation of forces are given in **Table 1.**

Table 1 Wave Parameters

S.No	Parameter	Definition	Formulae
1	Wave Length (L)	The distance between two consecutive crests or troughs	$L = \frac{gT^2}{2\pi}\tanh\frac{2\pi d}{L}$
2	Wave Celerity (C)	The velocity of the wave form with respect to the fluid in which it is travelling. It is also called phase speed.	$C = \frac{L}{T}$
3	Wave Number (k)	The spatial frequency of a wave measured in cycles per unit distance.	$k = \frac{2\pi}{L}$
4	Group celerity (C_g)	The speed at which the energy of the wave travels.	$C_g = \frac{1}{2}\left(1 + \frac{2kd}{\sinh 2kd}\right)\frac{L}{T}$

4 DECK LEVEL IDENTIFICATION

Waves entering the deck of an offshore structure will develop significant loads on it and the general practice is to set the deck above reasonably foreseeable wave conditions. Factors to be taken into account for setting the deck level **(Fig.2)** are extreme waves, tides, storm surge, foundation subsidence and interaction of large waves with structural components. The adoption of sufficient deck clearance is a tried and tested method within the offshore industry, and is known as 'air gap' approach.

An air gap of 1.5 m above the maximum water level needs to be considered for identifying deck level. An additional allowance may also be considered for sea level rise during the life of the structure based on the importance and location of the structure.

Minimum Deck Level = Water Depth with respect to Charted Datum (d_{cd}) + Maximum Tidal Range (TR) + Storm Surge (SS) + Maximum Wave Crest Height (η) + Minimum Air Gap (A_g) of 1.5m + Navigation clearence

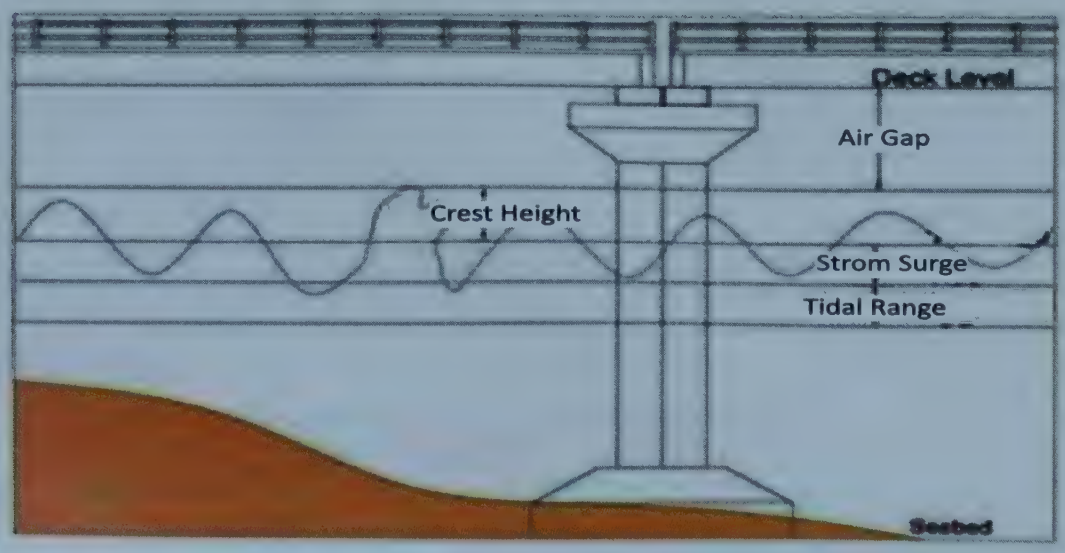


Fig.2 Identification of Deck Level

4.1 Tidal Range

Tides are the rise and fall of sea levels caused by the combined effects of the gravitational forces exerted by the Moon and the Sun, and the rotation of the Earth. Tidal Ranges for various locations in open coast along India are provided in **Appendix – II [9].**

4.2 Storm Surge

The level of ocean surface can change due to factors such as changes in atmospheric pressure and winds accompanying the passage of pressure systems (cyclones). This rising of the sea as a result of wind and atmospheric pressure changes associated with a storm is called storm surge. Storm surge values are highly site specific and hence statistical analysis of past data or detailed model studies need to be carried out. Storm Surge values for various locations in open coast along India are provided in **Annexure – III [9] (Based on 10, 50 & 100 years returns period).** These values can be used for preliminary design. However, for detailed design, detailed analysis of past data or detailed model studies shall be carried out as explained.

4.3 Wave Crest Height

Crest height of wave is the height of wave above still water level. Numerical methods using stream function theory are available to compute the wave crest height with good accuracy. However a simple approximation for the wave crest height is given below.

$$\eta = \frac{H}{2}\exp\left(\frac{2\pi}{L}\frac{H}{2}\right)$$

where H is the maximum wave height at the site and L is the wave length. The water level to be used for deriving the maximum wave height should be dcd + TR + SS.

5 WAVE TRANSFORMATION

Waves undergo a process of transformation when they move from deep water to shallow waters. These processes include refraction, shoaling, breaking and others. Assuming the seabed contours to be almost parallel and given the wave height at a deeper location, to estimate the wave height at a point of interest (shallower water), the following expression can be used:

$$H_i = H_0 K_s K_r$$

where, H_i is wave height at point of interest, H_0 is offshore wave height, K_s is shoaling coefficient, K_r is refraction coefficient.

5.1 Wave Refraction

The change in wavelength of a wave as it moves towards the shore changes the speed with which the wave travels along its crest. This process is called refraction and as a result the direction of the incoming wave at a shallow depth will be different from that of the direction at deeper depths. The direction at shallow waters (say, project site) can be found, provided the direction at deeper waters using the simple relation as given below,

$$\frac{\sin\theta}{C} = \frac{\sin\theta_0}{C_0}$$

Where θ and C are the wave direction and wave celerity respectively and the suffix '0' represents offshore water condition of the corresponding parameter.

The Wave Refraction Coefficient is given as, $K_r = \sqrt{\frac{\cos\theta_0}{\cos\theta}}$

5.2 Wave Shoaling

The propagation of wave to shallow water also involves increase in the wave height, called shoaling. It is caused by the fact that the group velocity, which is also the wave-energy transport velocity, changes with water depth. The shoaling coefficient can be obtained using the following relation,

$$K_s = \sqrt{\frac{C_{g_0}}{C_g}}$$

C_g is the group velocity and suffix '0' represents offshore water condition.

5.3 Wave Breaking

If the transformed wave height at the required location becomes so high such that the water column is unable to sustain the wave, the wave will break. Breaking of waves is a highly dynamic phenomenon which is difficult to predict or model. As a general rule, when the transformed wave height is more than 0.78 times the water depth (breaker index), the wave will break. Different formulas that include various parameters like wave steepness and bed slope can also be used to arrive at the breaker index [10]. If the structure is at such a location it has to be designed for breaking wave forces.

6 WAVE REGIME CLASSIFICATION

The wave forces on a structure depend on the wave parameters like water depth, wave height, wave period etc., and also on the characteristic dimension of the structure. The size of the structure determines the disturbance in the wave form caused by it. The bigger the structure the bigger is the disturbance. Since the physical interaction changes based on the structure size, different methods are to be employed for estimation of wave forces. The value of D/L, where D is the characteristic dimension of the structure submerged under water and L is the wavelength, is very useful for differentiating between the various methods. It is useful in classifying the problem into three "regimes". Generally the following three classification are given,

$D/L > 1$	-	Reflection regime
$0.2 < D/L < 1$	-	Diffraction regime
$D/L < 0.2$	-	Morison's regime

Different methods for wave force estimation are to be adopted for the different regimes. For normal bridge piers the interaction with waves will mostly fall in the Morison regime.

7 WAVE FORCES ON SMALL DIAMETER CIRCULAR MEMBERS

7.1 Non Breaking Waves

For estimating of non-breaking wave forces, the wave kinematic needs to be estimated using suitable wave theory, followed by Morison equation. The schematic is shown in **Fig.3.**

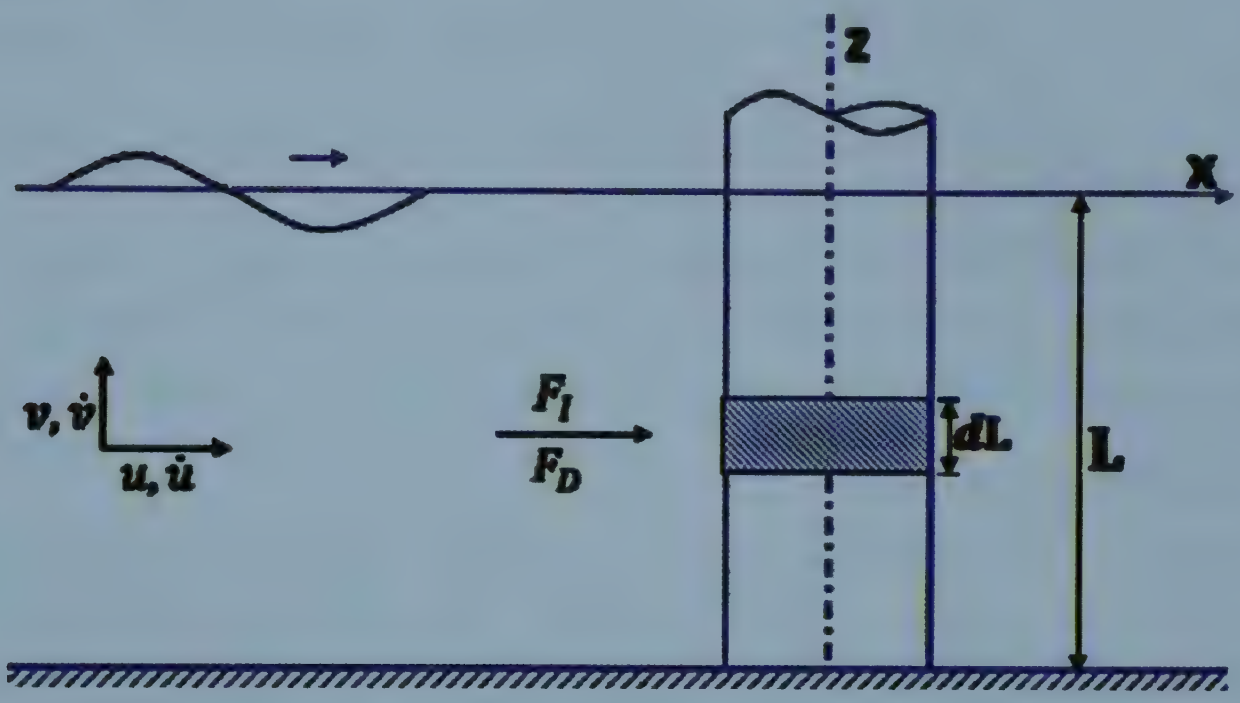


Fig.3 Non Breaking Wave Force Estimation

7.1.1 Wave Kinematics

The expressions for instantaneous water level, particle velocities and acceleration are derived from linear theory. At time t at a distance x from the wave crest and at a height z below still water level (z=0 at the still water level and z = -d at the seabed) the expressions are given below

$$u = \frac{\pi H}{T}\frac{\cosh[k(d+z)]}{\sinh(kd)}\cos(kx-\omega t)$$

$$v = \frac{\pi H}{T}\frac{\sinh[k(d+z)]}{\sinh(kd)}\sin(kx-\omega t)$$

$$\dot{u} = \frac{\pi H\omega}{T}\frac{\cosh[k(d+z)]}{\sinh(kd)}\sin(kx-\omega t)$$

$$\dot{v} = \frac{\pi H\omega}{T}\frac{\sinh[k(d+z)]}{\sinh(kd)}\cos(kx-\omega t)$$

where,

z is the height from still water level. z is 0 at still water level and negative below the still water level

u is the horizontal water particle velocity

v is the vertical water particle velocity

$\dot{u}$ is the horizontal water particle acceleration

$\dot{v}$ is the vertical water particle acceleration

d is the still water depth

H is the wave height

$L = \frac{gT^2}{2\pi}\tanh(kd)$ is the wave length

T is the time period

k is the wave number = $\frac{2\pi}{L}$

ω is the angular frequency = $\frac{2\pi}{T}$

Linear wave theory is normally assumed to be valid but in shallow waters where the ratio of depth to deepwater wavelength (d/L_o where, $L_o = \frac{gT^2}{2\pi}$) is less than 0.1 the wave form deviates significantly from the sinusoidal wave form of linear theory. Hence it may be necessary to go for higher order wave theories. Software packages such as ACES (automated Coastal Engineering System) developed by coastal engineering research center of the US Army Corps for Engineers (USACE) or similar softwares developed internationally can be used in such cases.

7.1.2 Morison Theory

The Morison equation is used for estimation of wave forces in conditions where D/L < 0.2. In the Morison equation the wave force is simplified as having two components, Drag component and Inertia component. These two components are out of phase with each other since the drag component is dependent on velocity and the inertia component dependent on acceleration. The Morison equation is given below,

$$F = F_D + F_I$$

where,

F – Total wave force normal to the axis of the member (kN)

F_D – Drag force component (kN)

$$F_D = \int_0^L \frac{1}{2} C_D \rho D u|u|\, dL;$$

F_I – Inertia Force component (kN)

$$F_I = \int_0^L C_M \rho \frac{\pi D^2}{4} \dot{u}\, dL:$$

where,

L is the submerged length of which dL is an elemental length (m)
C_D is the drag coefficient
C_M is the inertia coefficient
ρ is the density of water
D is the characteristic dimension of the structure
u is the horizontal water particle velocity
|u| is the absolute value of U
$\dot{u}$ is the horizontal water particle acceleration

The length L is governed by the instantaneous water level η when the member extends above the water level.

The coefficients C_D and C_M are based on structural parameters like size, shape and relative roughness and flow parameters like the Keulegan carpenter number and the Reynold's number.

For typical design situations the following values of drag and inertia coefficients can be used for unshielded circular cylinders (as per [6]):

Smooth	$C_D = 0.65$, $C_M = 1.6$
Rough	$C_D = 1.05$, $C_M = 1.2$

For a detailed discussion on the selection of drag and inertia coefficients [4] and [5]. It should be noted that the surface of the pier should be considered as rough when there is scope of marine growth. In addition, marine growth can increase the diameter of the structure and the modified diameter with expected marine growth should be considered for the design.

7.1.3 Influence of Currents

When there is current in addition to waves the Morison equation has to be changed accordingly. The wave kinematics has to be calculated based on the apparent wave period and then if the current is inline, the magnitudes are added to give the total velocity. For a non-collinear current, the component of current in-line with the wave is used. The modified equation is given below,

$$F = \frac{1}{2} C_D \rho D (U + V)|U + V| + C_M \rho \frac{\pi D^2}{4} \dot{U}$$

where V is the velocity of the current in m/s. A current in the wave direction tends to stretch the wave length, while an opposing current shortens it. As a result of this interaction the apparent wave period is different from the actual one. This modified wave period can be found using the procedure given in [6] and [7] and may be used in the calculation of wave kinematics.

7.2 Breaking Waves

Wave breaking directly on the structure, results in severe wave pressure depends on the nonlinear wave transformation over finite water depth. The schematic is shown in **Fig.4** and net breaking force, F_b in kN, acting at the water level can be determined as follows:

$$F_b = 0.5\gamma_w C_{D1} D H_b^{\,2}$$

where,

γ_w is the unit weight of water
D is diameter of the pier
H_b is breaking wave height ≈ 0.78 d
C_{D1} is drag coefficient for breaking waves,
= 1.75 for round piers
= 2.25 for square or rectangular piers

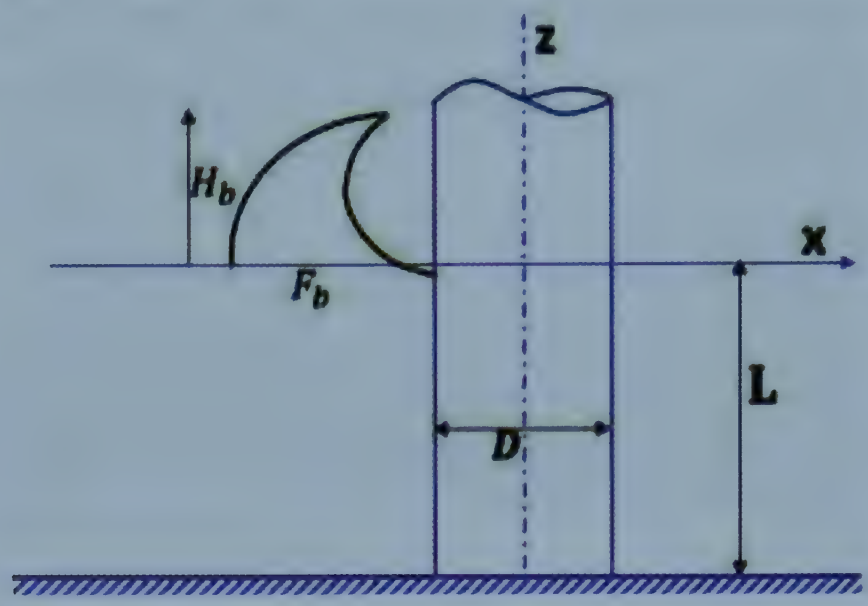


Fig.4 Breaking Wave Forces

8 WAVE FORCES ON LARGE DIAMETER MEMBERS

When the characteristic dimension of the structure is large when compared with wavelength i.e. 0.2 < D/L < 1 the condition falls under the diffraction regime. In the diffraction regime the structure alters the form of the incident wave over the area in its vicinity and the flow remains attached to the body of the structure, that is, flow separation is not expected. These factors necessitate an approach different from Morison's equation for the estimation of wave forces. The diffraction theory given in [3] is valid for linear cases but if there is significant nonlinearity involved in the interaction either physical modelling or numerical modelling shall be performed for higher accuracy.

9 WAVE FORCES ON REFLECTIVE STRUCTURES

Walls are structures that almost completely reflect the incoming wave. The forces exerted on these structures can be distinguished into Non-breaking, breaking and broken wave forces. Forces due to non-breaking waves are primarily hydrostatic. Broken and breaking waves exert an additional force due to the dynamic effects of turbulent water and the compression of entrapped air pockets. Dynamic forces may be much greater than hydrostatic forces and therefore structures located where waves break are designed for greater forces than exposed only to Non-breaking waves.

9.1 Goda's Method

The Goda method is quasi-static method that partly takes into account the impulsive breaking forces but is widely accepted and used for estimating wave forces on reflective structures. A description of the Goda method along with schematic **(Fig.5)** is given below:

$$\eta^* = 0.75(1 + \cos\beta)H_{max}$$

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{\frac{4\pi d}{L}}{sinh\left(\frac{4\pi d}{L}\right)}\right]^2$$

$$\alpha_2 = min\left\{\frac{d_b - d_s}{3d_b}\left(\frac{H_{max}}{d_s}\right)^2, \frac{2d_s}{H_{max}}\right\}$$

$$\alpha_3 = 1 - \frac{h'}{d}\left[1 - \frac{1}{cosh\left(\frac{4\pi d}{L}\right)}\right]$$

where, $d_b = 5H_s\tan\theta + h$

H_{max} is the maximum expected wave height or design wave height, d is the design water depth, L is the wavelength, d_s is the height of the structure submerged in water, H_s is the significant wave height.

Wave Pressures on the front of a vertical wall

$$P_1 = \frac{1}{2}(1 + cos\beta)(\alpha_1 + \alpha_2 cos^2\beta)H_{max}$$

$$P_3 = \alpha_3 * P_1$$

$$P_4 = \begin{cases} P_1 * \left(1 - \frac{h_c}{\eta^*}\right) & \eta^* > h_c \\ 0 & \eta^* < h_c \end{cases}$$

Where h_c is the height of structure above design water level, β is the angle between direction of wave approach and a normal to the plane of the structure.

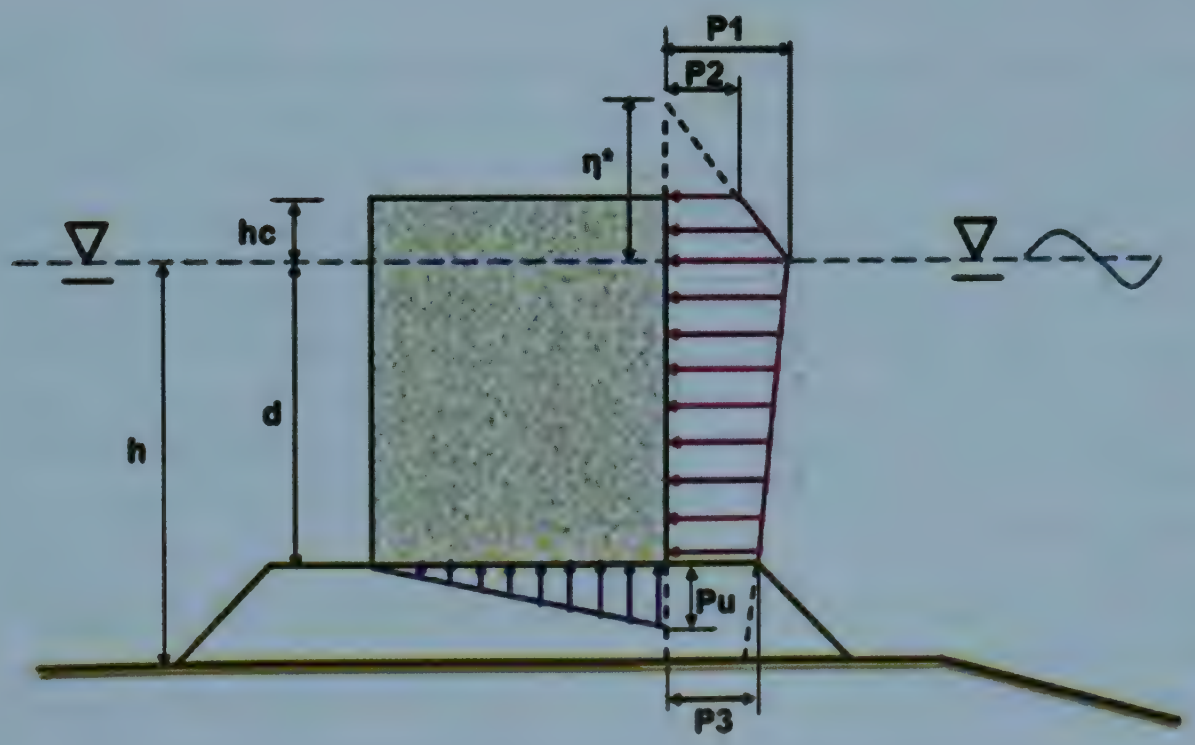


Fig.5 Pressure Distribution of Goda's Method

The uplift pressure acting on the bottom of the section is assumed to have a triangular distribution with toe pressure P_u given by

$$P_u = \frac{1}{2}(1 + cos\beta)\alpha_1\alpha_3\rho g \mathrm{H_{max}}$$

With the above formulas for wave pressure, the total wave pressure and its moment around the bottom of the section is calculated with the following equations.

$$P = \frac{1}{2}(P_1 + P_3)h' + \frac{1}{2}(P_1 + P_4)h_c^*$$
$$M_p = \frac{1}{6}(2P_1 + P_3)h'^2 + \frac{1}{2}(P_1 + P_4)h_c^* h' + \frac{1}{6}(P_1 + 2P_4)h_c^{*2}$$

In which, h_c^*=min$\{\eta^*, \mathrm{h_c}\}$

There are various modifications given to Goda's method detailed discussions on which can be seen in Coastal Engineering Manual, 2008. If the instantaneous impulsive pressures produced by wave breaking exactly at the face of the structure is to be considered in the design then Goda's method may under-predict the wave forces. In such a case Minikin's method as detailed in Shore Protection Manual, 1984 can be used.

10 NUMERICAL MODELLING FOR WAVE FORCES

Offshore structures, by virtue of its application, always exposed to the harsh sea environment and experience the impact loads due to wave breaking. So, wave-structure interaction is a classical problem in coastal and offshore engineering with an objective of estimation of wave diffraction around the structure, the wave forces or moments acting on the structures and the structural responses to the wave action [1]. The best design of offshore structure will arrive based on accurate prediction of above parameters. Study of wave loads on the structure is complex due to complexity in wave structure interaction [2]. Because the structure in water interrupts the propagation of waves and the flow field associated with waves depends on both wave conditions and geometry of the structure.

Analytically, Morison's equation is widely used for the estimation of wave loads in design of offshore and coastal structures for structures with length in wave direction relatively small compared with the wavelength, such as piles, braces and legs of jacket platforms. The semi-empirical equation was based on the assumption that wave kinematics is not significantly affected by the presence of structures. For offshore structures, application of Morison's equation is limited to structures with diameter smaller than 1/5 of wavelength. In Coastal Engineering Manual, the limit on diameter of structure is reduced to 1/20 of wavelength. In case of bridge super structure and complicated geometry like wind turbine monopile, the interaction between structure and waves should not be neglected because of larger size. Instead, conducting full or laboratory scale experiments is better option to study the complex wave structure interaction. This may cost and time consuming process and may not be possible to test large number of cases for different parameters.

Numerical studies are one possible way to get reliable interaction with exercise of extreme care in terms of numerical constants, velocity scales, length scale, turbulence characteristics and more over accurate modelling of sea state (normal waves, breaking waves etc.). There are two categories of numerical modeling of wave structure interaction, namely, the potential flow model and Navier–Stokes based model.

1. Potential Flow Model

These models are capable of calculating wave interaction with large bodies where the viscous effect is negligible. Diffraction analysis needs to be performed to compute wave induced pressure on the surface of superstructures. Wave loads can then be computed by integrating pressure on wetted area of super structures.

2. Navier -Stokes Based Model

Navier–Stokes equation (NSE) is applicable to wider range of problems when viscous and turbulence effects are significant. These models can apply using Finite Element Method (FEM), Finite Difference Method (FDM) and Finite Volume Method (FVM).

11 LOAD COMBINATIONS

Partial Safety Factor for Verification of Structural Strength			
	Ultimate Limit State		
Loads	Basic Combination	Accidental Combination	Seismic Combination
(1)	(2)	(3)	(4)
Permanent Loads:			
Dead Load, Snow load if present, SIDL except surfacing			
a) Adding to the effect of variable loads.	1.35	1.0	1.0
b) Relieving the effect of variable loads.	1.0	1.0	1.0
Surfacing:			
a) Adding to the effect of variable loads.	1.75	1.0	1.0
b) Relieving the effect of variable loads.	1.0	1.0	1.0
Prestress and Secondary effect of prestress			
a) Back fill weight.	1.50	1.0	1.0
Earth pressure due to back fill			
a) Leading Load.	1.50	-	1.0
b) Accompanying Load.	1.0	1.0	1.0
Variable Loads:			
Carriageway line load and associated loads (bracking, tractive and centrifugal forces) and			
Pedestrian Live Load:			
a) Leading Load.	1.5	0.75	0
b) Accompanying Load.	1.15	0.2	0.2
c) Construction Live Load.	1.35	1.0	1.0
Wind during service and construction			
a) Leading Load.	1.50	-	-
b) Accompanying Load.	0.9	-	-
Live load Surcharge (as accompanying load)	1.2	0.2	0.2
Erection effects.	1.0	1.0	1.0
Accidental Effects:			
a) Vehicle Collision or Barge impact or impact due to floating bodies.	-	1.0	-
Seismic Effects			
a) During service.	-	-	1.0
b) During construction.	-	-	0.5
Hydraulic Loads (Accompanying Load):			
a) Water Current Forces.	1.0	1.0	1.0
b) Wave Pressure	1.0	1.0	1.0
c) Hydrodynamic effect	-	-	1.0
d) Buoyancy	0.15	0.15	0.15

Storm Surge

Same (Wind- 1.0)

——

Same

Wave Forces:
a) Leading Forces – 1.5
b) Accompanying – 0.9

Storm Surge:
a) During Service – 1.0
b) During Construction – 0.5

EXAMPLE PROBLEM 1: Wave Force on a Pile using Morison's Equation

Estimation of wave forces on 1 m diameter pile located in 8 m water depth with a shoreline orientation of 135° degrees with respect to North. The necessary data is taken for Location 12.

SOLUTION:

Step 1 Wave Parameters at 25 m Water Depth

The wave height for a specific return period and the angle of approach with respect to North at a water depth of 25 m can be taken from the table given in **Appendix**. In this case, the value of the wave height at 25 m depth for 20 years return period (H_{25}) is 5.2 m with an angle of 255° with respect to North.

Step 2 Wave Length, Celerity and Group Celerity at Required Location

Wave Period (T) of 12 s is considered for example

Parameter	Formulae	Reference Depth (25 m)	Required Depth (8 m)
Wave Length (L)	$L = \frac{gT^2}{2\pi}\tanh\frac{2\pi d}{L}$	165.96 m	102.33 m
Wave Number (k)	$k = \frac{2\pi}{L}$	0.0378	0.0614
Celerity (C)	$C = \frac{L}{T}$	13.83 m/s	8.53 m/s
Group Celerity (C_g)	$C_g = \frac{1}{2}\left(1 + \frac{2kd}{\sinh 2kd}\right)\frac{L}{T}$	10.96 m/s	9.16 m/s

Step 3 Transformed Wave Height at required Water Depth

The approach angle with respect to shore normal at reference depth, θ_{25} =

= Wave Angle - Shoreline Angle – 90°
= 255 – 135 – 90 = 30°

The approach angle with respect to shore normal at required water depth of 8 m can be estimated using the relation:

$$\frac{\sin\theta_8}{C_8} = \frac{\sin\theta_{25}}{C_{25}}$$

θ_8= 18°

The transformed wave height at required water depth can be estimated,

$$H_8 = H_{25}\sqrt{\frac{C_{g,25}}{C_{g,8}}}\sqrt{\frac{\cos\theta_{25}}{\cos\theta_8}}$$

$H_s = 5.43$ m

Check for Wave Breaking

Wave Height to Water Depth Ratio $= H_s / d$ $= 5.43 / 8$

$= 0.68$ < 0.78

Hence, waves will not break.

Step 4 Wave Kinematics

For calculating wave force using Morison's equation, the horizontal water particle velocity (u) and acceleration ($\dot{u}$) need to be determined. Assuming linear theory the expression for velocity, u and acceleration, $\dot{u}$ at an instant time t at a specific height y below still water level measured in the negative direction (still water level is considered zero level) are given below:

$$u = \frac{\pi H}{T} \frac{\cosh[k(y+d)]}{\sinh(kd)} \cos(\omega t)$$

$$\dot{u} = \frac{\pi H \omega}{T} \frac{\cosh[k(y+d)]}{\sinh(kd)} \sin(\omega t)$$

Where H is the wave height, T is the time period and d is the water depth. ω ($2\pi/T$) is the angular frequency and k ($2\pi/L$) is the wave number.

For the current problem, H = 5.43 m (transformed wave height at 8 m water depth)

T= 12s; k =$2\pi/L$=0.0614; ω=$2\pi/T$=0.5236; d=8 m

The level y varies from 0 at still water level to –d i.e. -8 at the seabed. The total depth of 8m should be split into multiple segments at an interval of, say 1 m, and the velocity and acceleration are to be determined for each level. After finding the velocity and acceleration the Morison equation can be used to determine the force per unit length (kN/m) at each segment.

Step 5 Wave force on Pile

Assuming a rough cylindrical structure, the drag and inertia coefficients can be taken as C_d=1.05 and C_M=1.2. The Morison equation (sec 7.1.2) will be used for finding forces. The variation of the force through a wave cycle (0° to 360°) at each level (0 to -8 m) in the water column is given in the table below :

Depth\Phase	0	30	60	90	120	150	180	210	240	270	300	330	360
0	5.25	4.73	2.68	1.58	0.06	-3.15	-5.25	-4.73	-2.68	-1.58	-0.06	3.15	5.25
-1	4.98	4.50	2.58	1.54	0.09	-2.96	-4.98	-4.50	-2.58	-1.54	-0.09	2.96	4.98
-2	4.75	4.32	2.49	1.50	0.11	-2.81	-4.75	-4.32	-2.49	-1.50	-0.11	2.81	4.75
-3	4.57	4.16	2.42	1.47	0.13	-2.69	-4.57	-4.16	-2.42	-1.47	-0.13	2.69	4.57
-4	4.42	4.04	2.36	1.45	0.15	-2.59	-4.42	-4.04	-2.36	-1.45	-0.15	2.59	4.42
-5	4.30	3.94	2.31	1.43	0.16	-2.51	-4.30	-3.94	-2.31	-1.43	-0.16	2.51	4.30
-6	4.22	3.88	2.28	1.42	0.17	-2.46	-4.22	-3.88	-2.28	-1.42	-0.17	2.46	4.22
-7	4.18	3.84	2.26	1.41	0.18	-2.43	-4.18	-3.84	-2.26	-1.41	-0.18	2.43	4.18
-8	4.16	3.82	2.26	1.41	0.18	-2.42	-4.16	-3.82	-2.26	-1.41	-0.18	2.42	4.16
TOTAL FORCE (kN)	36.13	32.95	19.18	11.71	1.11	-21.24	-36.13	-32.95	-19.18	-11.71	-1.11	21.24	36.13

EXAMPLE PROBLEM 2: Wave and Current Force on a Pile Using Morison's Equation

Estimation of wave forces on 1 m diameter pile located in 8 m water depth with a shoreline orientation of 135° degrees with respect to North. A steady current of velocity of 1m/s in the direction of the wave is present at the location. The necessary data is taken for Location 12.

SOLUTION:

In comparison with Example Problem 1 there is an additional current present which has to be considered.

Step 1 to Step 4 will be the same as Problem 1.

Step 5 Wave Force on Pile

In Step 5, the modified Morison as given below should be used (section in 7.1.3).

$$F = \frac{1}{2} C_D \rho D (U + V)|U + V| + C_M \rho \frac{\pi D^2}{4} \dot{U}$$

where, V is the current velocity at a particular level in the water column which has to be added with the wave particle velocity at that same level. The current velocities vary with site condition and it is taken to be constant throughout the water column for this example.

Assuming a rough cylindrical structure, the drag and inertia coefficients can be taken as C_d=1.05 and C_M=1.2. The variation of the force through a wave cycle (0° to 360°) at each level (0 to -8 m) in the water column is given in the table below :

Depth\Phase	0	30	60	90	120	150	180	210	240	270	300	330	360
0	9.15	8.18	4.90	2.12	1.20	-0.77	-2.43	-2.35	-1.54	-1.04	2.16	6.60	9.15
-1	8.79	7.88	4.75	2.08	1.19	-0.67	-2.24	-2.21	-1.48	-1.00	2.09	6.34	8.79
-2	8.49	7.62	4.63	2.04	1.17	-0.58	-2.09	-2.08	-1.43	-0.97	2.02	6.12	8.49
-3	8.24	7.41	4.52	2.01	1.16	-0.51	-1.97	-1.98	-1.39	-0.94	1.97	5.94	8.24
-4	8.04	7.25	4.44	1.99	1.15	-0.46	-1.87	-1.91	-1.36	-0.91	1.93	5.80	8.04
-5	7.89	7.12	4.38	1.97	1.15	-0.42	-1.80	-1.85	-1.33	-0.89	1.90	5.69	7.89
-6	7.78	7.03	4.33	1.96	1.14	-0.39	-1.75	-1.80	-1.31	-0.88	1.87	5.61	7.78
-7	7.71	6.97	4.30	1.95	1.14	-0.37	-1.72	-1.78	-1.30	-0.87	1.86	5.56	7.71
-8	7.69	6.95	4.29	1.94	1.14	-0.36	-1.71	-1.77	-1.30	-0.87	1.86	5.55	7.69
TOTAL FORCE (kN)	65.36	58.84	35.94	16.02	9.27	-3.96	-15.51	-15.67	-11.02	-7.41	15.65	47.13	65.36

EXAMPLE PROBLEM 3: Breaking Wave Forces on A Pile

Estimation of wave forces on 1 m diameter pile located in 4 m water depth with a shoreline orientation of 135° degrees with respect to North. The necessary data is taken for Location 12.

SOLUTION:

Step 1 Wave Parameters at 25 m Water Depth

The wave height for a specific return period and the angle of approach with respect to North at a water depth of 25 m can be taken from the table given in Appendix. In this case, the value of the wave height at 25 m depth for 20 years return period (H_{25}) is 5.2 m with an angle of 255° with respect to North.

Step 2 Wave Length, Celerity and Group Celerity at Required Location

Wave Period (T) of 12 s is considered for example.

Parameter	Formulae	Reference Depth (25 m)	Required Depth (4 m)
Wave Length (L)	$L = \frac{gT^2}{2\pi}\tanh\frac{2\pi d}{L}$	165.96 m	73.77 m
Wave Number (k)	$k = \frac{2\pi}{L}$	0.0378	0.0818
Celerity (C)	$C = \frac{L}{T}$	13.83 m/s	6.39 m/s
Group Celerity (C_g)	$C_g = \frac{1}{2}\left(1 + \frac{2kh}{\sinh 2kh}\right)\frac{L}{T}$	10.96 m/s	6.18 m/s

Step 3 Transformed Wave Height at required Water Depth

The approach angle with respect to shore normal at reference depth, taken for Location 12.

θ_{25} = Wave Angle - Shoreline Angle – 90°

= 255 – 135 – 90 = 30°

The approach angle with respect to shore normal at required water depth of 4 m can be estimated using the relation;

$$\frac{\sin\theta_4}{C_4} = \frac{\sin\theta_{25}}{C_{25}}$$

θ_4= 16.37°

The transformed wave height at required water depth can be estimated,

$$H_4 = H_{25}\sqrt{\frac{C_{g,25}}{C_{g,4}}}\sqrt{\frac{\cos\theta_{25}}{\cos\theta_4}}$$

H_4 = 6.58 m

Check for Wave Breaking

Wave Height to Water Depth Ratio = H_4 / d = 6.58 / 4

= 1.645 > 0.78

Hence, waves will break.

Step 4 Estimation of Breaking Wave Forces

Breaking wave forces are determined using the formula given below (refer sec 7.2)

$$F_b = 0.5\gamma_w C_D D H_b^{\,2}$$

where,

γ_w is the unit weight of water

D is diameter of the pier

H_b is breaking wave height $\approx 0.78\,d$

C_D is drag coefficient for breaking waves.

= 1.75 for round piers

$F_b = 0.5 * 9.8 * 1.75 * 1 * (0.78*4)^2$
$= 83.47$ kN

This load is to be applied as a lateral point load acting at the design water level.

EXAMPLE PROBLEM 4: Minimum Deck Level Fixation for Bridges

Estimation of minimum deck level of the bridge for two different conditions. The input conditions are taken from Example Problems 1 & 2.

SOLUTION:

As per Section 4, the minimum deck level is given by,

Minimum Deck Level = Water Depth with respect to Charted Datum (d_{cd}) + Maximum Tidal Range(TR) + Storm Surge(SS) + Maximum Wave Crest Height(η) + Air Gap(Ag)nce depth,

	Problem 1 (8 m depth)	**Problem 2 (4 m depth)**
Water Depth, d_{cd}	8 m	4 m
Maximum Tidal Range (TR)	1 m	1 m
Storm Surge (SS)*	2 m	2 m
Max. Wave crest Height (η)	$=\frac{H}{2}\exp\left(\frac{2\pi}{L}\frac{H}{2}\right)$ (sec 4.3) = 3.01 m	Wave breaking. Consider entire wave height to be crest. η = 0.78*d = 0.78*4 = 3.12 m
Air Gap (A_g)	1.5 m	1.5 m
Minimum deck level	15.51 m	11.62 m

*- Storm surge is assumed to be same at both water depths here. However, in reality storm surge is highly site specific and may be estimated using statistical or numerical models.

The base of the deck slab should be at least the minimum deck level value above the seabed.

EXAMPLE PROBLEM 5: Wave Force on a Reflective Stricture using Goda's Method

Estimation of wave forces on a reflective structure located in 8 m water depth with a shoreline orientation of 135° degrees with respect to North. The necessary data is taken for Location 12.

SOLUTION:

Step 1 Wave Parameters at 25 m Water Depth

The wave height for a specific return period and the angle of approach with respect to North at a water depth of 25 m can be taken from the table given in Appendix. In this case, the value of the wave height at 25 m depth for 20 years return period (H_{25}) is 5.2 m with an angle of 255° with respect to North.

Step 2 Wave Length, Celerity and Group Celerity at Required Location

Wave Period (T) of 12 s is considered for example.

Parameter	Formulae	Reference Depth (25 m)	Required Depth (8 m)
Wave Length (L)	$L = \frac{gT^2}{2\pi}\tanh\frac{2\pi d}{L}$	165.96 m	102.33 m
Wave Number (k)	$k = \frac{2\pi}{L}$	0.0378	0.0614
Celerity (C)	$C = \frac{L}{T}$	13.83 m/s	8.53 m/s
Group Celerity (C_g)	$C_g = \frac{1}{2}\left(1 + \frac{2kh}{\sinh 2kh}\right)\frac{L}{T}$	10.96 m/s	9.16 m/s

Step 3 Transformed Wave Height at required Water Depth

The approach angle with respect to shore normal at reference depth, θ_{25}

= Wave Angle - Shoreline Angle – 90°
= 255 – 135 – 90 = 30°

The approach angle with respect to shore normal at required water depth of 8 m can be estimated using the relation;

$$\frac{\sin\theta_8}{C_8} = \frac{\sin\theta_{25}}{C_{25}}$$

$$\theta_8 = 18°$$

The Transformed Wave Height at required water depth can be estimated,

$$H_8 = H_{25}\sqrt{\frac{C_{g,25}}{C_{g,8}}}\sqrt{\frac{\cos\theta_{25}}{\cos\theta_8}}$$

H_8 = 5.43 m

Step 4 Wave Force

Wave forces for reflective structures, i.e. dimension of structure is large in comparison with the wavelength (see Section Goda's method on the document), are estimated using Goda's method. Considering the structure height to be 11 m, the crest elevation (difference between design water depth and structure height) is 3 m. Assume a seabed slope (M) of 1 in 25. The maximum impact on the structure will happen when the wave breaks at 5 times Hs away from the structure. The depth at this point, h_b can be found as below :

$$h_b = d + \left(5 * \frac{H_{max}}{1.8} * M\right) = 8.46\ m$$

The wave height to depth ratio is (5.43/8.46 = 0.641) less than the critical value of 0.78 and hence the wave will not break. In case if the wave height to depth ratio exceeds 0.78 the waves may break and the design wave height should be taken as 0.78d.

A summary of the design inputs is given below.

Design Inputs

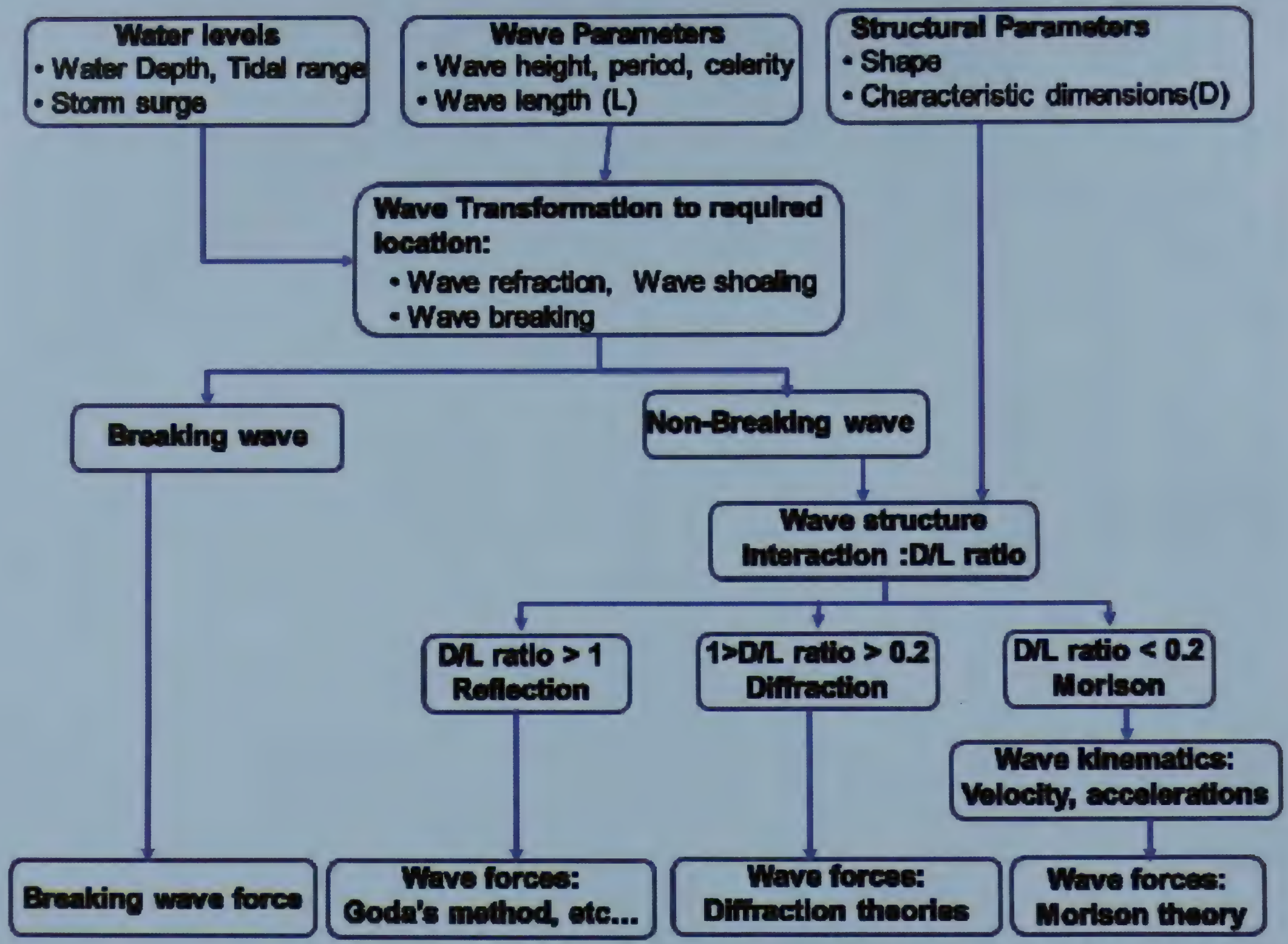
Density of Sea water	=	1.025	tonne/m^3
Design Water depth, h_s=h'=d	=	8	m
Design Wave height, H_{max} or H_{design} (from appendix and transformation)	=	5.43	m
Wave Period, T	=	12	s
Wave length, L (from (1))	=	102.33	m
Assuming a structure height, h_w	=	11	m
Crest elevation, h_c= d - h_{st}	=	3	m
Wave number, k (2π/L)	=	0.06	
Seabed slope, M (assuming bed slope of 1 in 25)	=	0.04	
Water depth at a distance of 5 H_s seaward of the structure, h_b (d+(5* H_{max}/1.8*M))	=	8.46	m

The wave pressures are calculated for per m run and a model calculation is given below:

Coefficients for the Wave Pressure

α_1	$0.6+0.5*((2*k*h_s)/(\sinh 2*k*h_s))^2$	=	0.97	
α_2	$\min\{(h_b-d/3*h_b)*((H_{design}/d)^2), (2*d/H_{design})\}$	=	0.01	
α_3	$1-\{((h_w-h_c)/h_s)*[1-(1/(\cosh(2\pi*h_s/L)))]\}$	=	0.89	
α^*	$\alpha 2$	=	0.01	
β	Angle of incidence of waves(Assuming 0 for worst case)	=	0	deg
$\lambda_1=\lambda_2=\lambda_3$		=	1	
η^*	$0.75*(1+\cos\beta)*\lambda_1*H_{design}$	=	8.14	
Pressure Components				
Pressure at Water Level, P_1	$0.5*(1+\cos\beta)*(\lambda_1*\alpha_1+\lambda_2*\alpha^*\cos 2\beta)*\rho_w*g*H_{design}$	=	53.3	kPa
Pressure at top of Structure, P_2	$(1-h_c/\eta)*P_1$ for $\eta^*>h_c$, 0 for $\eta^* \le h_c$	=	33.6	kPa
Pressure at Bottom, P_3	α_3*P_1	=	47.4	kPa
Uplift Pressure, P_u	$0.5*(1+\cos\beta)*\lambda_3*\alpha_1*\alpha_3*\rho_w*g*H_{design}$	=	46.9	kPa
h_c^*	$\min(\eta^*,h_c)$		3	
Total Pressure, P	$0.5*h_c^**(P_1+P_2)+0.5*(P_1+P_3)*(h_w-h_c)$	=	534	kN/m run
Moment of Pressure, M_P	$(1/6)*(2*P_1+P_3)*h'^2+0.5*(P_1+P_2)*h'*h_c+(1/6)*(P_1+P_2)*hc^2$	=	2816	kN.m/m run

"FLOW CHART : ESTIMATION OF WAVE FORCES"



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APPENDIX-I

Wave Parameters at various Locations*

Location	Lon	Lat	Maximum wave height in m for different return period [years]					Wave Angle
			2	5	20	50	100*	
1	87.4110	21.0000	5.2	6.3	7.6	8.2	8.7	180 ^{o}
2	86.5290	20.0000	5.2	6.7	9.1	10.6	11.9	195 ^{o}
3	84.7400	18.9800	3.9	5.4	7.6	8.9	9.9	180 ^{o}
4	83.7560	18.0000	3.9	5.2	7.1	8.4	9.3	180 ^{o}
5	82.4260	17.0000	3.7	4.7	5.6	6.1	6.5	135 ^{o}
6	81.3230	16.0000	3.3	4.8	6.9	8	8.9	150 ^{o}
7	80.1930	15.0000	3.3	4.8	6.3	7.3	7.9	135 ^{o}
8	80.3340	14.0000	3.9	5.6	7.3	8.4	9.0	135 ^{o}
9	80.3570	13.0000	3.9	5.6	8	9.3	10.3	150 ^{o}
10	79.9690	12.0000	3.5	5	6.7	7.6	8.4	150 ^{o}
11	79.9350	11.0000	3.7	4.5	5.2	5.6	5.9	150 ^{o}
12	77.5000	8.0290	4.5	4.8	5.2	5.4	5.4	255 ^{o}
13	76.4200	9.0000	4.5	5	5.8	6.1	6.5	270 ^{o}
14	76.0720	10.0000	4.7	5.4	6.5	7.1	7.5	270 ^{o}
15	75.7270	11.0000	4.7	5.4	6.1	6.7	7.1	270 ^{o}
16	75.0970	12.0000	5.2	6	6.9	7.3	7.6	270 ^{o}
17	74.6770	13.0000	5.2	6	6.9	7.3	7.7	270 ^{o}
18	74.3860	14.0000	5.6	6.5	7.4	8	8.4	270 ^{o}
19	73.8660	15.0000	6.1	7.3	8.4	9.1	9.6	255 ^{o}
20	73.3890	16.0000	6.3	7.4	8.7	9.3	9.7	255 ^{o}
21	73.1830	17.0000	6.1	7.3	8.6	9.3	9.9	255 ^{o}
22	72.8350	18.0000	6	6.9	8.2	8.7	9.3	255 ^{o}
23	72.5700	19.0000	6	6.9	8.2	8.9	9.6	255 ^{o}
24	72.5090	20.0000	5	6	7.1	7.8	8.4	240 ^{o}
25	71.5000	20.4230	5.8	6.5	7.4	8	8.4	240 ^{o}
26	70.1220	21.0050	5.8	6.9	8.4	9.3	10.2	240 ^{o}
27	69.1250	22.0000	5.6	6.7	8.6	10	11.2	240 ^{o}

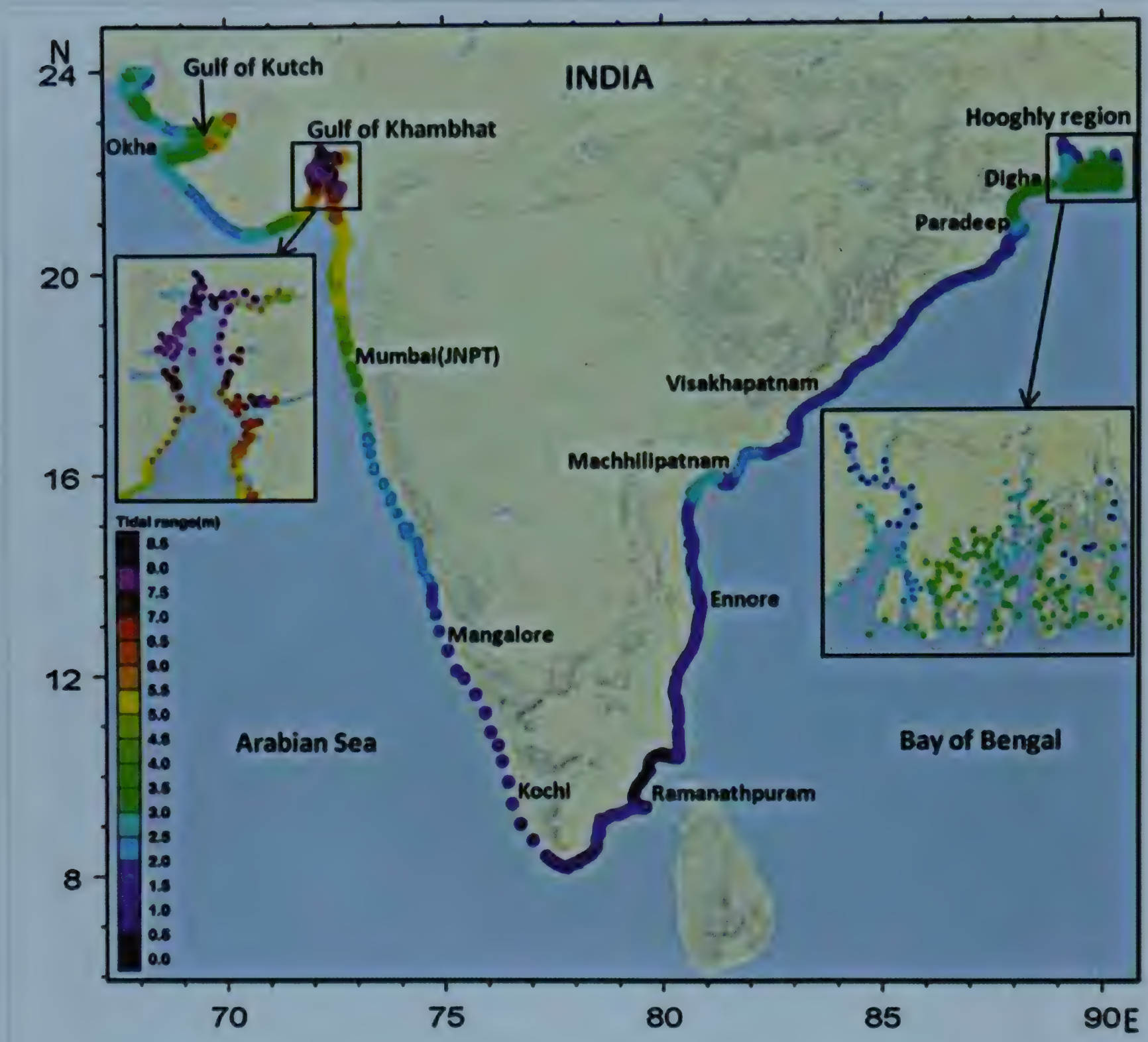
*Refer Wave atlas for Indian Coast by NIOT

APPENDIX-IA

Locations of Wave Parameters

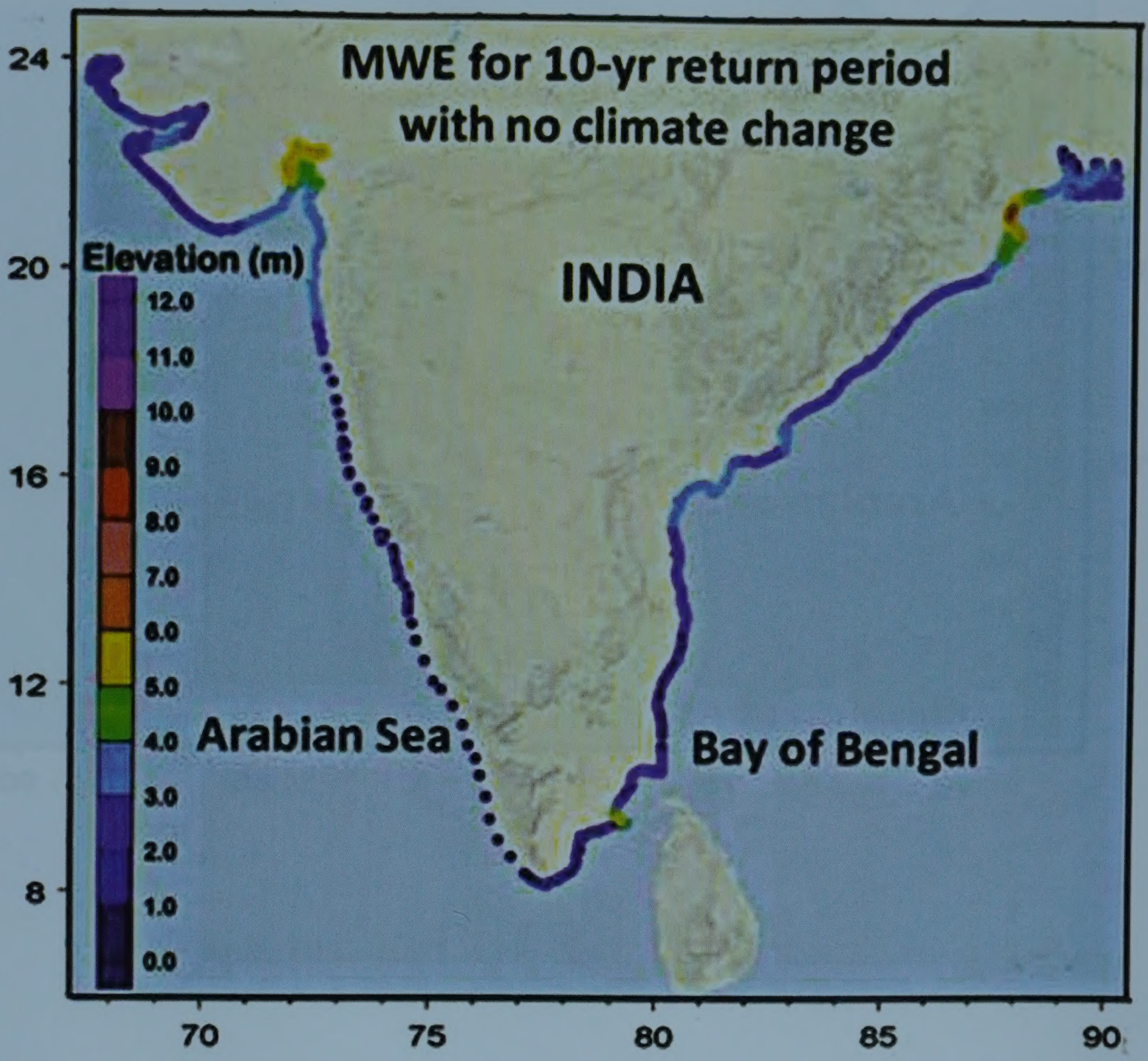
APPENDIX-II

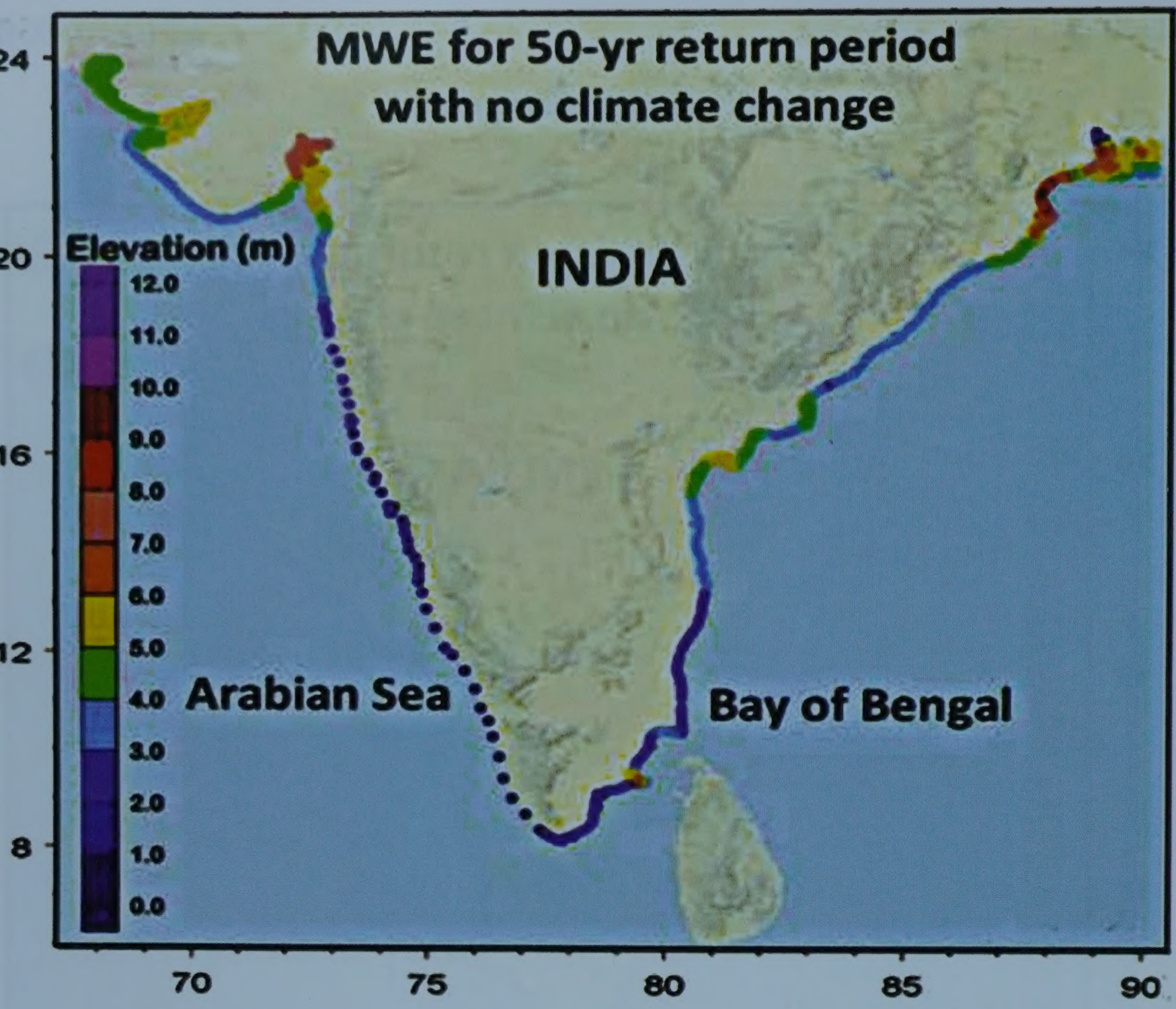
Tidal Range along India Coast



APPENDIX-III

Storm Surge




MWE for 50-yr return period
with no climate change
INDIA
Elevation (m)
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Arabian Sea
Bay of Bengal
24
20
16
12
8
70
75
80
85
90

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